737

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MORRO BAY STEAM ELECTRIC PLANT

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POWER DIVISION

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MORRO BAY STEAM ELECTRIC PLANT

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SYNOPSIS

The principal features of the Morro Bay Steam Electric Plant of particular interest to the civil engineer are as follows:

1) A condenser cooling water system that utilizes a surge chamber.
2) A 24-inch diameter fuel oil supply line extending 4400 feet into the ocean.
3) The first industrial application of sea water evaporators for the production of fresh water in the nation.
4) A continuous mat foundation under the main power building.
5) A 450 feet high reinforced concrete stack incorporating latest concepts in seismic design.
6) A dynamic design for turbine generator foundations.

INTRODUCTION

During the post-war period of 1946 to 1954 the Pacific Gas and Electric Company has experienced a tremendous load growth in the San Joaquin Valley. The peak load produced primarily by agricultural pumping in this area has increased 150 per cent as compared with a system growth of 105 per cent during the same period.

It soon became apparent that additional capacity would be required by 1955 to meet this growth in load. The economics of fuel costs and transmission losses coupled with the all important requirements for cooling water were the main factors in choosing Morro Bay as the location of this source of capacity.

Location

Morro Bay is situated on the Pacific Ocean half way between San Francisco and Los Angeles and 13 miles northwest of San Luis Obispo. The plant site is a 140 acre tract at the north end of the bay between the beachline and State Highway 1 near famous Morro Rock. Railroad facilities were available with a railhead at Camp San Luis Obispo on State Highway 1 about 10 miles from the steam plant site.

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General Design

The plant arrangement is based on an ultimate installation of eight complete units with two control centers each controlling four units. The initial installation will consist of two 156,250 kw reheat turbine generators and two 1,135,000 pounds per hour boilers. The turbine inlet conditions will be 1800 psi pressure, 1000°F temperature and a reheat temperature of 1000°F. The fuel will be oil with provision for future conversion to the use of pulverized coal and petroleum coke if necessary. The first unit will go into operation in April of 1955 and the second unit in March of 1956. The total cost of the initial two units is estimated at $44,300,000.

The principal engineering features of the plant are outlined in Figure 1. Power is to be generated at 18,000 volts and stepped up to 230,000 volts with 3 single phase transformers for each unit. Two 230,000 volt circuits will transmit the major portion of the power to Gates substation in the San Joaquin Valley near Coalinga, about 65 miles northeast of Morro Bay; however, power will also be transmitted at 110,000 volts over one circuit to San Luis Obispo substation for local distribution.

Main Condenser Cooling Water System

A major problem in the design of a plant of this size is that of pumping large quantities of water to the main condensers for cooling purposes and returning this heated water to its source without recirculation.

The cooling water system of this plant is ideal in this respect in that recirculation is virtually impossible. 200,000 gpm or 450 sec. ft. will enter the intake structure and be pumped by four mixed flow type pumps through four 700 foot lengths of 54"I.D. reinforced concrete pipe to the condensers. The discharge from each condenser will then be collected in a common 7' x 10' concrete tunnel and conveyed for 2850 feet to the base of Morro Rock and discharged into the Pacific Ocean.

In addition to the long discharge tunnel to Morro Rock, the following alternatives were investigated:

1) Discharge into the ocean at a point where the marine fuel line enters the ocean, or
2) Discharge back into Morro Bay.

The beach to the north of Morro Rock is a part of the clam beds occurring along the central California coast which precludes discharging directly into the surf by either an open channel or buried conduit. The only solution for this alternative would be a buried conduit across the beach and a marine pipe line of sufficient length so as not to create a hazard to the public from a standpoint of fishing and bathing. An economic comparison of this system with that of discharging at Morro Rock showed a savings in favor of Morro Rock due, in a large part, to the high construction costs of the marine portion of the former system.

The second alternative, that of discharging into Morro Bay, involved a study of temperatures of the water in the bay and their effect on the vacuum in the condensers and hence the efficiency of the plant.

Morro Bay has a tidal prism between mean lower low water and mean higher high water of 8800 acre feet. The temperature of the water in the summer varies from 54°F at flood tide to 62°F at ebb tide. During the winter these same temperatures are practically constant at 50°F. Inasmuch as the

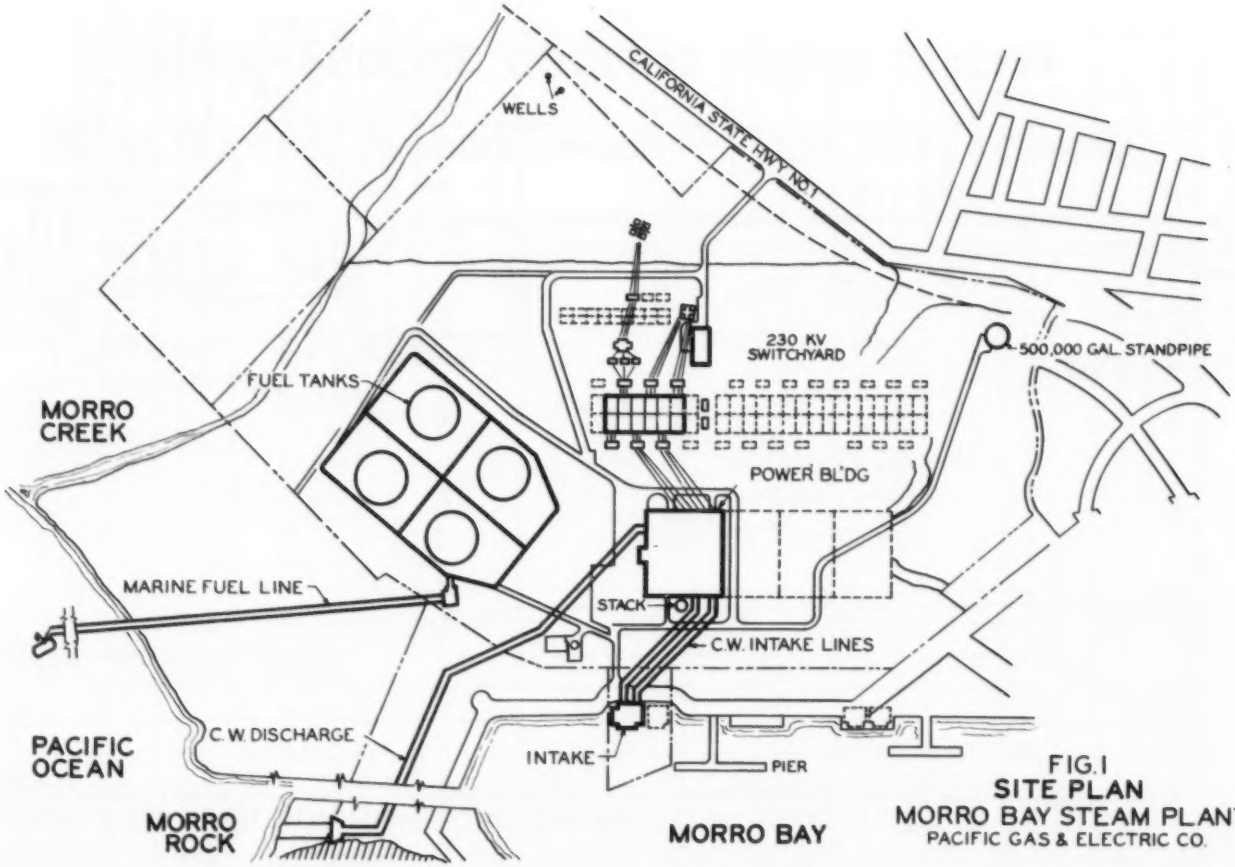

FIG.1
SITE PLAN
MORRO BAY STEAM PLANT
PACIFIC GAS & ELECTRIC CO.

737-3

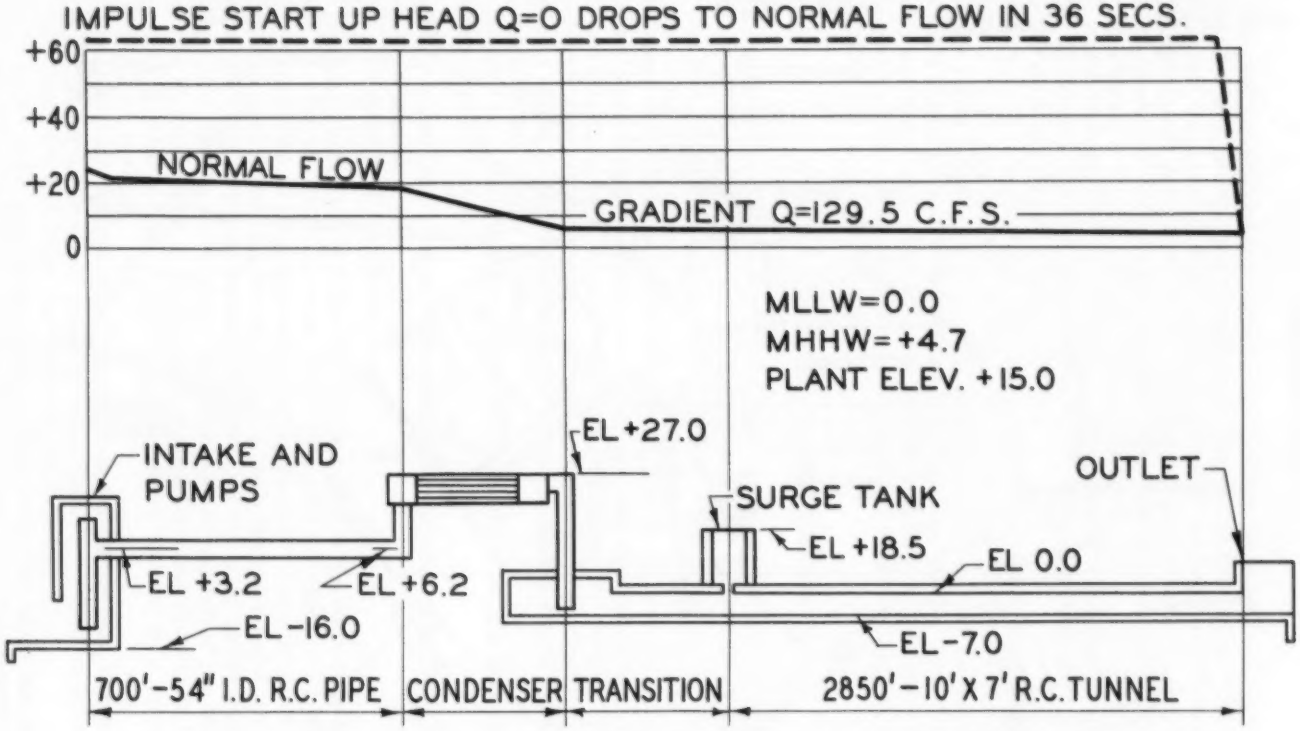

FIG.2 - PROFILE COOLING WATER SYSTEM

737-4

major demands for power will occur during the summer, tidal conditions at that time were the main concern.

While passing through the main condensers the cooling water temperature is raised 15°F at maximum load. The problem then was to determine how much temperature rise would occur in the bay by adding 450 sec. feet of 15° warmer water. Several assumptions were made as to the degree of mixing of the discharge water with the tidal water. These assumptions were as follows:

1) A complete mixing of the water during the tidal cycles under consideration and resulting in a uniform heating of the tidal water.
2) The discharge water remaining stratified and channelized and under this same tidal condition flowing directly into the intake structure thereby producing a maximum rise in temperature.
3) The discharge water remaining stratified and channelized and flowing past the intake structure thereby producing no temperature increase.

A study of the tidal currents and the topography of the bay indicated that the actual conditions lie between assumptions 2 and 3 and were quite closely represented by the first assumption.

Based on this assumption of complete mixing of the discharge water with the water of the bay the following conclusions were reached as to variations in temperature.

No. of Condensers Operating	Harbor Temperature Rise From Recirculation	Percent of Time
2	0°-1°	66
	1°-2°	20
	2°-5°	14
8	0°-5°	75
	5°-10°	12
	10°-15°	8
	15°-20°	5

Any increase in temperature of the cooling water will impair the condenser vacuum with a corresponding increase in fuel consumption. The maximum capacity of the turbine generator will also decrease with an inferior vacuum.

Based on a 2°-5° rise for two units and 14 per cent of the time the additional capitalized fuel costs and capacity losses for summer conditions and peak load demand would be $270,000. The additional cost of discharging at Morro Rock due to the increased conduit and related structures was $200,000 or $70,000 less.

Other conditions than the purely economic had to be considered. A fairly large oyster industry is operating at the lower or southerly end of Morro Bay. Consultation with biologists indicated that any increase in water temperature over the normal variation might affect the oyster culture. Should any damage occur it might be difficult to prove that the warm discharge was not a contributing factor. The possible danger to small boats within the harbor due to the exit velocities from the discharge also had to be considered.

Surge Tank

Although the long discharge tunnel to Morro Rock was the most desirable from a standpoint of operation and economy, it presented a serious hydraulic problem. (see Fig. 2).

The invert of the discharge tunnel is at elevation -7 and the crown is at elevation zero referred to mean lower low water datum. With this arrangement the tide is effective the entire length of the tunnel and accomplishes two important functions:

1) It provides an effective seal for the siphon or discharge leg of the condenser.
2) It provides a maximum regain of head on the siphon leg of the condenser under all tidal conditions.

The disadvantage of this arrangement occurs under conditions of low tide and after a shut down when the condensers are empty. A start up of one circulating water pump may cause a serious water hammer in the 54'' reinforced concrete intake line during the acceleration of the large mass of water in the discharge tunnel.

Under normal conditions the mixed flow type pump will start with an impulse head of 53 feet with Q = 0. Some thirty-six seconds later it will operate at a uniform flow of 130 sec. ft. with a total developed head of 19 feet. An investigation of what occurs in this thirty-six second time interval revealed that when the condenser started to spill, the flow in the intake lines would be 105 sec. ft. and the flow in the discharge tunnel would be zero. Some six and one-half seconds later the flow in the intake lines would drop to 82 sec. ft. and the flow in the tunnel would be 22 sec. feet which would fill the system for the first time. At this instant water hammer would occur in the intake lines. The velocity would then drop from 5.16 feet per second to 1.41 feet per second causing a 282,000 foot pound shock to the system and developing a static head of 367 feet.

It was not felt that it would be economical to design for a pressure rise of that magnitude; however, there were two other solutions to the problem:

1) Install butterfly valves at the circulating water pump in the intake line in an effort to throttle the flow and allow the pump flow to gradually increase until the system was in equilibrium, or
2) Place a surge tank in the system to absorb the energy produced by the water hammer.

Of the two solutions, the surge tank was more economical and more desirable from the standpoint of reliable operation in that there were no mechanical parts to fail when their operation was most needed.

The surge tank is a restricted orifice type with a 48'' diameter orifice in the crown of the 10' x 7' tunnel. The tank is 20'-0'' x 8'-0'' and extends 18'-6'' above the tunnel and allows a maximum surge of 9.0 feet.

Thermal Mussel Control (See Fig. 3)

Mussels are indigenous to the Pacific Ocean coastal waters, and their growth in the circulating water system could be quite serious if allowed to flourish without control. Experience at other plants indicates that mussels will cling to the walls of the circulating water conduits up to depths of 6 inches, thereby materially reducing their capacity.

It is proposed to control the growth of mussels at this plant by recirculating the warmed water between the condensers and the intake structure and allowing the temperature to rise excessively for short periods of time. Experience at other plants has shown that by raising the temperature to about 110^{0} for a period of 20 minutes every three weeks the mussels can be killed

and thereby loosened from the walls of the system and flushed through the condensers into the discharge water. The frequent application of this operation is necessary in order to prevent their growth beyond a point where they might plug the condenser tubes. The exact temperature, duration and frequency will have to be determined by operational experience.

This method of combating the growth of mussels is not new to the industry; however, it is believed that improvements have been made at this plant over other methods of handling the problem. Normally a cross connection is made just behind the pumps which confines the mussel control to the pipe lines. With the addition of the motor operated sluice gates in front of the traveling screens, the mussel control has been extended to include the entire intake structure. It is believed that this system will eliminate lengthy shutdowns to clean out the screen and pump wells.

To "de-mussel" Unit #1 the following procedure is to be followed:

1) Open condenser cross-over valve inside the condenser.
2) Stop circulating water pump #1.
3) Close condenser discharge valve #1.
4) Regulate reverse flow with condenser discharge valve #2.
5) Close intake gate #1.
6) Wash screen with hot screen wash pump.
7) Operate spare auxiliary cooling pump from Unit #2 to discharge.

The procedure may be reversed by stopping circulating water pump #2.

Fuel Oil System (Fig. 4)

The fuel oil delivery and storage system consists of a marine anchorage, 4483' of marine pipe, 1258' of land pipe and 4 - 168,000 barrel storage tanks.

The system is designed to receive 8000 barrels per hour of 600 SSF viscosity oil. Initially 200 SSF viscosity oil will be delivered at about 135°F temperature.

The marine anchorage 4400 feet offshore provides 5 - 10 ton permanent anchors. There are two anchors amidship and three anchors aft. The forward anchorage is provided by using the ship anchors. Each of these permanent anchors is located about 800 feet from the ship and is attached to a mooring buoy with 400 feet of chain. Each buoy consists of a 6'-6'' OD x 11'-0'' tank and provided with a quick release hook to fasten the ship mooring lines. Buoys are also provided to mark the ends of the pipe and hose and a radar reflector buoy is provided for navigational assistance to the anchorage during foul weather.

The anchorage is located in the prevailing currents in such a manner that the ship can set its forward anchors and a crew in a small launch can fasten the lines to the 3 mooring buoys aft. With the ship approximately in position with the fore and aft anchors, springing lines are then made fast to the mooring buoys amidship.

Delivery of oil is accomplished by picking up the end of the 200' long x 12'' diameter sea hose from the ocean bottom by means of the chain attached to its marker buoy and connecting it to the ship pumps.

The marine portion of the line is a 24'' OD - 1/2'' wall pipe with 3/4'' of somastic coating. In order to provide a negative buoyancy of 10-15 pounds, a 1-1/4'' gunite coating was added. The land portion of the line is a 24'' OD - 3/8'' wall pipe with 3/4'' somastic coating but with the gunite coating omitted. The marine portion of the line is cathodically protected against corrosion.

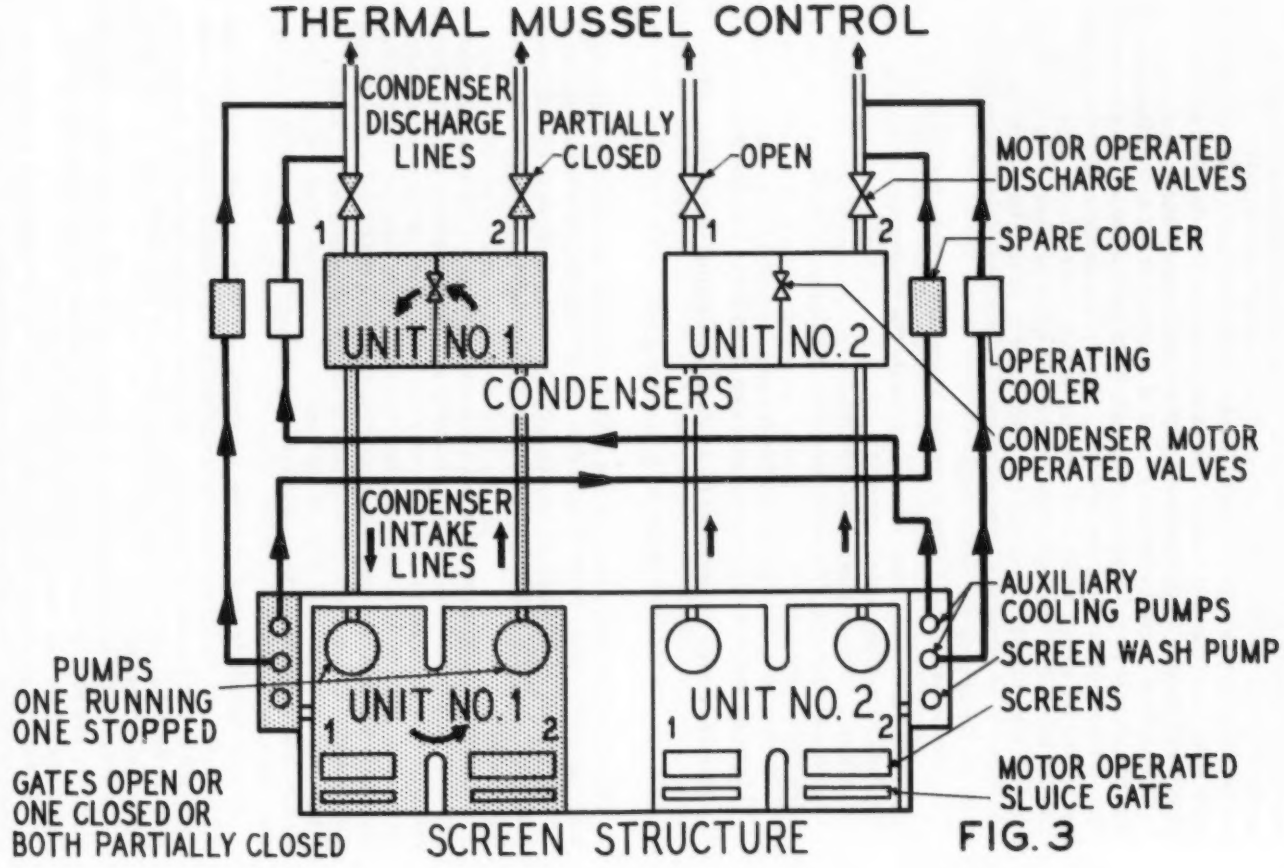
THERMAL MUSSEL CONTROL
CONDENSER DISCHARGE LINES
PARTIALLY CLOSED
OPEN
MOTOR OPERATED DISCHARGE VALVES
SPARE COOLER
UNIT NO. 1
UNIT NO. 2
CONDENSERS
OPERATING COOLER
CONDENSER MOTOR OPERATED VALVES
CONDENSER INTAKE LINES
AUXILIARY COOLING PUMPS
SCREEN WASH PUMP
PUMPS ONE RUNNING ONE STOPPED
SCREENS
GATES OPEN OR ONE CLOSED OR BOTH PARTIALLY CLOSED
MOTOR OPERATED SLUICE GATE
SCREEN STRUCTURE

FIG. 3

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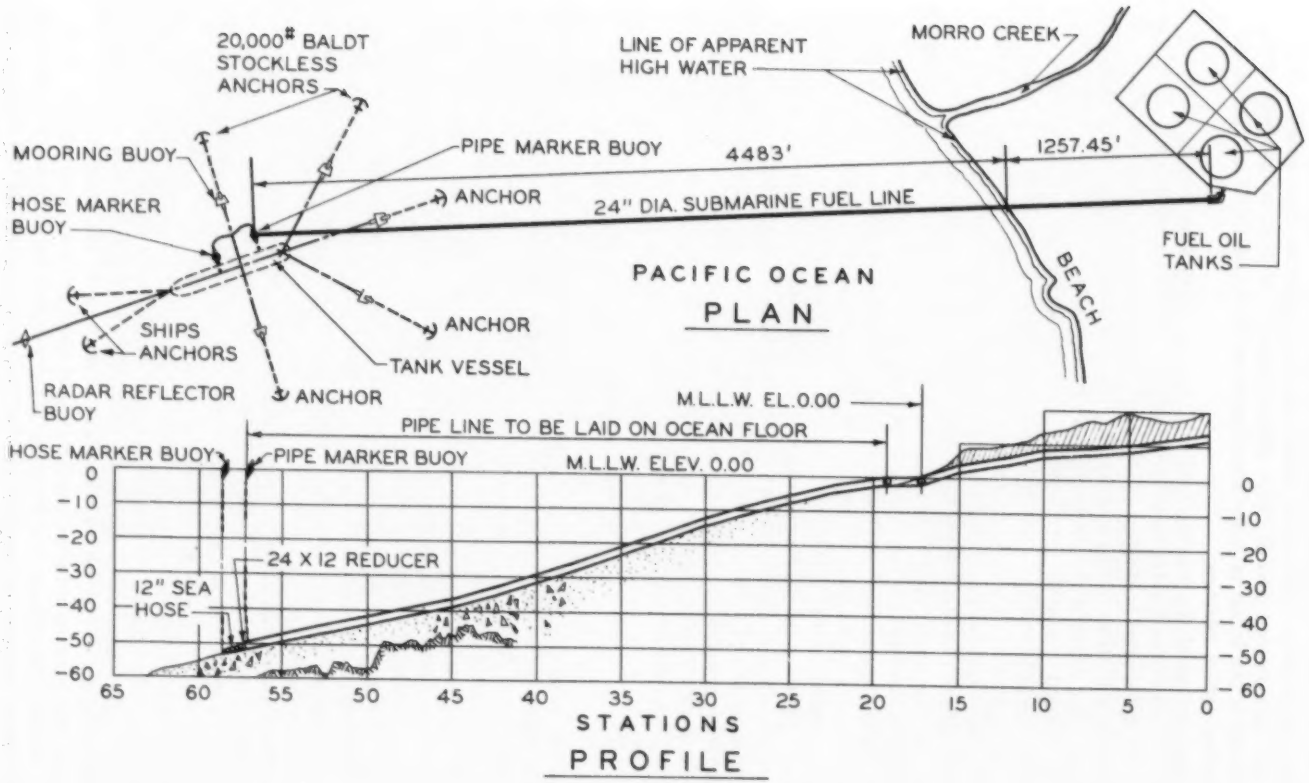

FIG. 4 - FUEL OIL SYSTEM

737-9

The fuel oil tanks are located above plant grade by 8' allowing oil to flow to the plant by gravity. All future tanks will be located at plant grade. Each tank is 195' in diameter and 32' high with a capacity of 168,000 barrels each. Temperature in the tanks is maintained between 100° and 120°F by means of steam recirculation heaters.

The installation of the marine fuel oil line was quite spectacular from the standpoint of coordination between land and sea construction forces. The pipe was initially delivered in lengths of 39 feet. These lengths were welded on the beach into 5 strings of approximately 820 feet each and a shorter string of 380 feet. The joints were somastic and gunite covered. The 24 x 12 reducer was welded to the 380 foot section and then mounted on a structural steel sled. (See Figure 5). A barge with a 60 ton winch, anchored in the mooring area, provided a 1-1/4'' pulling cable which was attached to the lugs provided behind the reducer. Side boom tractors then picked up the first string and with the barge pulling and the tractors pushing the section was launched into the surf. After the first string was launched an additional string was welded to the first string and in turn launched. A total of 12 side boom tractors were required for each string. Communication between the barge and land forces was maintained by ship to shore radio and coordination of land forces was accomplished by visual hand signals. After launching started the joints between sections were only welded and somastic coated. Wood battens were substituted for the gunite coating at these joints to protect the somastic coating and to provide a smooth surface for the pulling operation. The total operation was performed in 15 hours and 30 minutes. The average pulling time for each section was 30 minutes while the welding and somastic coating of the joints averaged 3 hours and 15 minutes each.

Erosion Control

When the property was first purchased the area to be occupied by the fuel oil tanks consisted of high sand dunes rising above the plant elevation by as much as 30 feet. This condition presented not only a problem from the standpoint of wind erosion but of utilizing the material to construct dikes to enclose the tanks for protection against spillage or breaks in the tanks. Several solutions to the problem were studied ranging from removing the dune sand entirely and constructing fire walls of precast and prestressed concrete to constructing the dikes of dune sand and guniting the slopes to prevent erosion.

The method finally adopted was ideal in that it was just as effective and more economical than any other method heretofore adopted at the company's other steam plant sites.

The dikes were constructed of the dune sand by grading the area down to about 3 feet below the final grade of the tank foundation and then rebuilding it to final grade by controlled compaction. Compaction was accomplished with a 50 ton pneumatic rubber tired roller to a field density of 95 per cent at optimum moisture content of the material. A strict control over the compaction was made by making tests at 20 foot intervals over the tank base area and 50 foot intervals in the dike areas. Relative compaction was determined by using the State of California, Division of Highways, standard methods.

During the interim between completion of the dike construction and wind erosion control treatment, the field forces protected the finished slopes from wind or water erosion by maintaining the moisture content of the surface particles to such an extent that they remained in a cohesive state. This was accomplished with rainmaker pipe equipped with fixed mist nozzles and

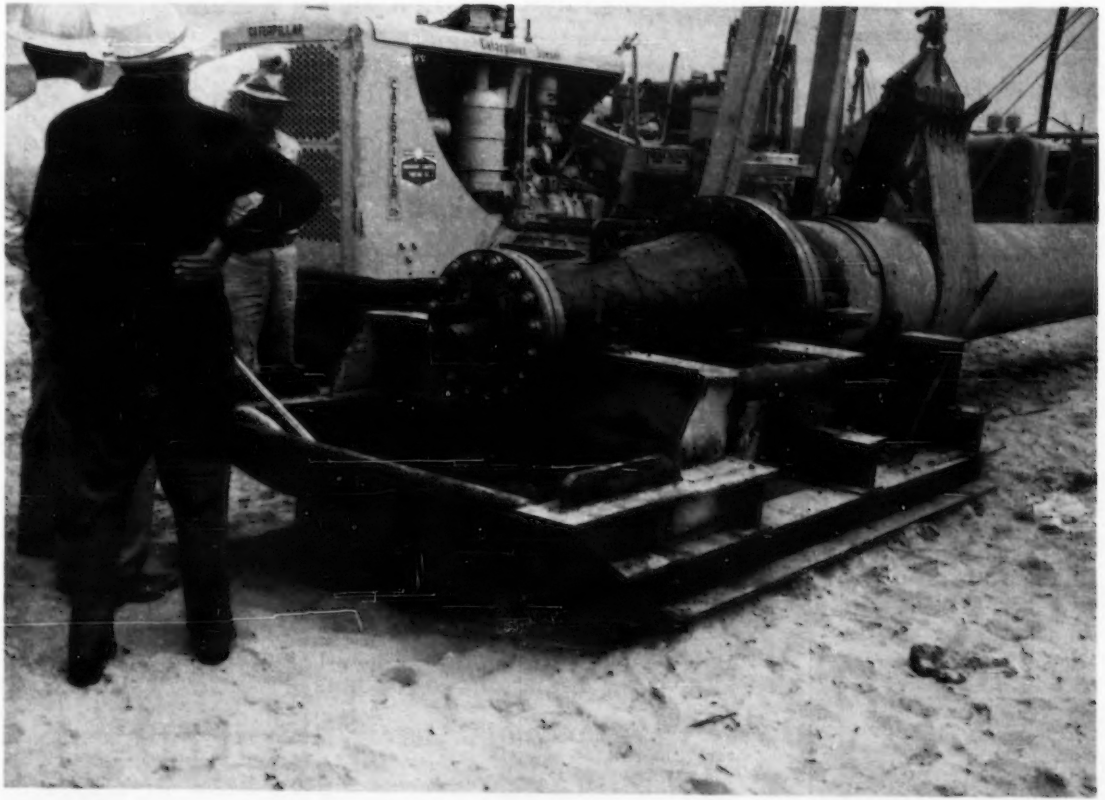

FIG. 5

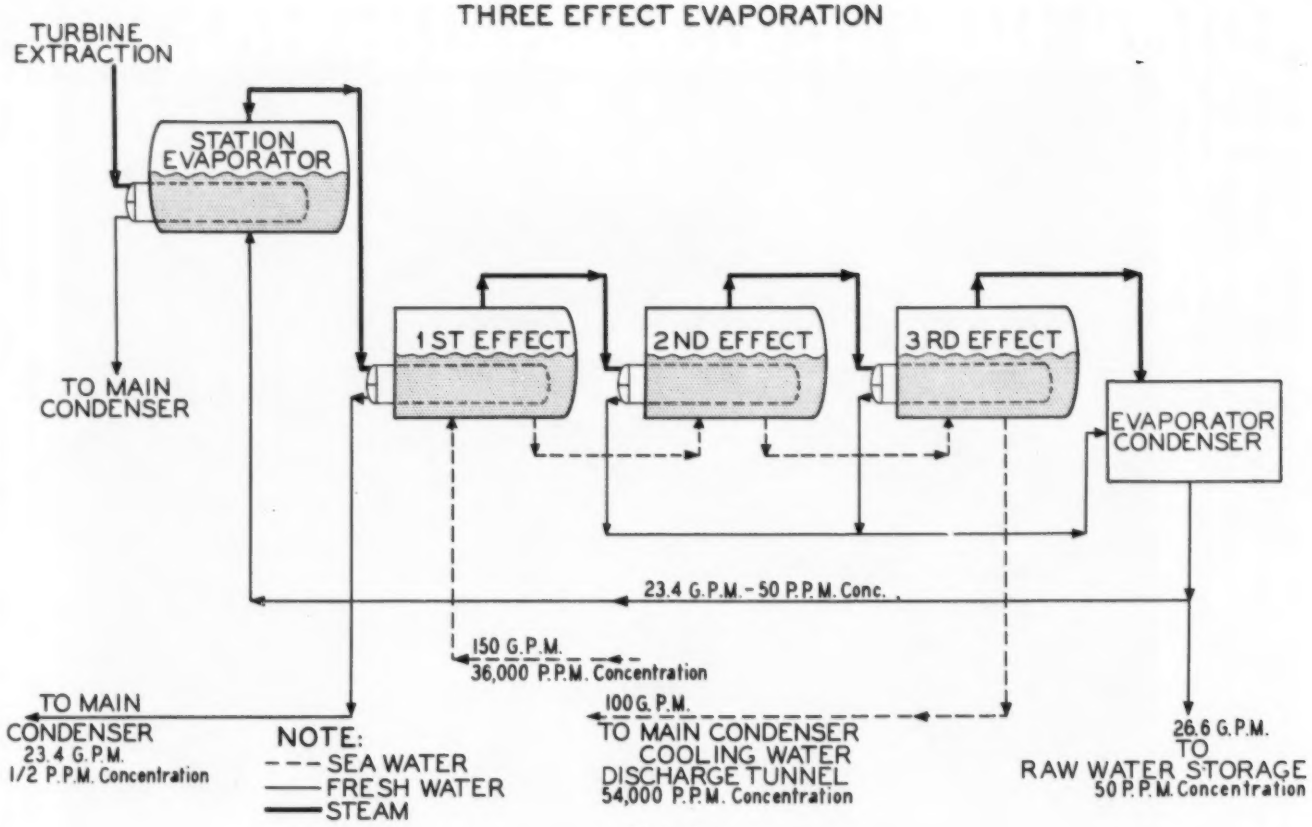
THREE EFFECT EVAPORATION
TURBINE EXTRACTION
STATION EVAPORATOR
1ST EFFECT
2ND EFFECT
3RD EFFECT
EVAPORATOR CONDENSER
TO MAIN CONDENSER
23.4 G.P.M. - 50 P.P.M. Conc.
150 G.P.M.
36,000 P.P.M. Concentration
100 G.P.M.
TO MAIN CONDENSER
23.4 G.P.M.
1/2 P.P.M. Concentration
NOTE:
SEA WATER
FRESH WATER
STEAM
TO MAIN CONDENSER COOLING WATER DISCHARGE TUNNEL
54,000 P.P.M. Concentration
26.6 G.P.M.
TO
RAW WATER STORAGE
50 P.P.M. Concentration

FIG. 6

installed along the top of the dikes. Care was taken so as not to concentrate the water in any given place and cause channeling and sloughing. The construction forces were also permitted to maintain these slopes during this period by applying straw to the surfaces at the rate of two to four tons per acre in thin layers and tamping the straw into sand by means of a modified sheep foot roller. If these methods did not maintain the slopes they were also allowed to add more straw and cover the straw with chicken wire mats laid flat on the ground and held in place by stakes. As it turned out the weather conditions were such that the latter two methods were not necessary.

Final erosion control was accomplished by planting "ice plant" (Mesembryanthemum Edule) on the dike slopes. The plant is native to the area and was actually obtained by thinning existing growths on the site.

The area was prepared by first broadcasting commercial pelleted fertilizer over the area at the rate of 10 pounds per 1000 square feet several days before planting. Rice straw was then spread over the area at the rate of 6 tons per acre on slopes of 5 to 1 or steeper and 4 tons per acre on flatter slopes. The straw was incorporated into the sand by use of a modified tamping roller consisting of thin lugs protruding 6'' from the roller, 6'' wide and 1/2'' thick instead of the usual sheep foot. This imbedded the straw and did not pull it out as the lug was withdrawn.

Cuttings of ice plant about ten inches long were planted through the straw and into the ground at intervals of 12'' on centers on the slopes and 18'' on centers on the flat areas.

The second application of fertilizer was made at the same rate about two months after planting was completed.

The nurseryman contracting the job was then required to maintain the growth for a period of ninety days after initial planting. During this period he watered the plants and replaced any that did not appear to be taking hold.

About thirteen acres were planted in this manner and after a year in service the results have been extremely satisfactory.

Fresh Water Supply

The fresh water requirements for the plant were determined to be about 80 gallons per minute per unit. The largest single items are 25 gpm for boiler makeup and 25 gpm for vertical pump bearing lubrication; however, salt water can be used for the latter item with a fresh water standby. The remaining 30 gpm consists of drinking and sanitary water; water for cracking the scale from the tubes in the evaporators and for flushing; and general utility such as water for flushing the boiler tubes during maintenance periods. The initial requirements for fresh water were set at 160 gpm for the initial two unit installation and 640 gpm for the ultimate installation of 8 units.

Complete dependability of the source was of a paramount importance both in quantity and quality especially from the standpoint of the water furnished for boiler makeup.

The sources available for investigation were as follows:

1) Upper Salinas River Dam.
2) Local wells.
3) Effluent from the Sewage Treatment Plant of the Morro Bay Sanitary District.
4) Impounding local runoff.
5) Sea water evaporators.

Upper Salinas River Dam

The Upper Salinas River Dam is located about 10 miles east of San Luis Obispo and is owned by the federal government and operated by the San Luis Obispo County Flood Control and Water Conservation District. The dam at the present time stores water for five public agencies. The City of San Luis Obispo and Camp San Luis Obispo have a joint allocation and are the largest users. Their needs are served by a joint pipeline that terminates at Chorro Creek Reservoir at Camp San Luis Obispo. To supply fully the ultimate plant needs would require a 12''pipe line about eleven miles long. Further investigation into priorities and allocations for storage indicated that a firm supply of water could not be assured.

Local Wells

Historic data on the wells in the area indicates that they are subject to deterioration both as to yield and quality of water. Shortages of ground water occurred during the recent and preceding series of dry years and some sea water has encroached into the ground water basins along the central coast. There were two wells on the property with an estimated combined yield of 250 gpm when the property was purchased. In view of the historical background, it did not seem advisable to rely on these wells as a principal source of water but it was felt that they could be used as a standby. A later experience during the early periods of construction justified this decision. At the time both wells were being continuously pumped to provide water for a concrete batch plant and other miscellaneous construction needs. One well went dry and the yield per foot of draw down greatly decreased in the other. Since then a third well has been drilled with a total combined yield of 200 gpm with the remaining well. A series of observation wells has been placed around these wells consisting of one and one half inch perforated pipe. With intermittent pumping the yield per foot of draw down has remained constant and the observation wells indicate that the seasonal ground water table is remaining consistently high. Monthly salinity tests of the water in the observation wells has not yet indicated salt water intrusion under the present conditions; however, the past year has been a fairly good rainfall year. During a dry year and low ground water table salt water intrusion could occur if the wells were pumped excessively.

Effluent from Sewage Treatment Plant

The Morro Bay Sanitary District, which includes the towns of Cayucos to the north, Morro Bay on the south and the intervening areas, has completed a sewage treatment plant situated on the property just to the north and adjacent to the steam plant site. This installation is a Clarifier and Filter type plant. An investigation indicated that the quantity of effluent could supply four steam units but a supply for our ultimate installation of 8 units would be doubtful. The quality of the effluent from a plant of this type is such that the detergents in the water would cause excessive foaming in the evaporators and restrict the chemical action of the water softener in the steam plant's own water treatment installation. The presence of ammonia from the high organic content in the effluent would cause excessive corrosion of the brass tubing in the evaporators. In the event of a breakdown in the sewage treatment no guarantee could be made of a continuous source of treated effluent. In view of the above objections this source could not be considered reliable.

Impounding Local Runoff

One of the most promising sources of fresh water was the impounding of local runoff. The local annual rainfall varies from 30 inches in the coastal range east of Morro Bay to 16 inches in Morro Bay. Although not abundant in runoff there were several small streams that could furnish the ultimate requirements of 1300 acre feet per year.

What seemed to be the most economical site for a dam was located on the Toro Creek water shed about 3 miles north and one mile east of the steam plant. The water shed above the dam was 14 square miles.

No long term runoff records were available for Toro Creek; however, a gaging station was established before the winter rains of 1952. The nearest comparable stream with gaging records was Arroyo Grande Creek about 25 miles south of Morro Bay with a drainage area of approximately 106 square miles.

The straight ratio of drainage areas shows that Toro Creek drains only 13.5 per cent of area of Arroyo Grande; however, by comparing areas of average normal rainfall and relating runoff to rainfall it appeared that Toro Creek should flow about 20 per cent the rate of Arroyo Grande. Comparison of actual measurements during the winter of 1952-1953 showed that the assumption of 20 per cent was on the conservative side.

A mass curve was plotted for the entire period of record based on the above assumption which indicated that a storage of 3,700 acre feet would be required to yield our ultimate demands of 1300 acre feet per year.

Material was available within a half mile of the site to construct an earth fill type dam. The dam would be 90 feet high requiring approximately 200,000 cubic yards of fill and creating a storage reservoir covering 175 acres. Consideration of the possibility of flash floods in the area indicated a 7000 sec. ft. side channel spillway would be required. Locating a dam at this site would also require a 4 mile pipe line to the steam plant, relocating 2 miles of county road and relocating existing oil transmission pipe lines in the storage reservoir area in addition to the purchase of land.

Sea Water Evaporators

While this process has been common aboard ships, it is believed that the use of sea water evaporators for producing fresh water for a stationary steam plant would be the first industrial application of this system in the nation.

Each steam generating unit will require a set of triple effect evaporators designed to produce 50 gallons per minute. This capacity was required to provide for both boiler make up and utility water. Since about half of the water produced from sea water evaporators need not be of the purity required for boiler makeup, a purity of 50 ppm total solids was specified. A part of this water would then be redistilled in the station evaporators to produce the required purity of less than 1 ppm for boiler makeup.

Fig. 6 shows a simplified outline of the three effect evaporation cycle. Each of these three evaporators consists of a 5/8'' thick welded copper bearing steel shell 6'-0'' OD x 20'-0'' long. The lower half of the shell contains 290 one inch diameter aluminum brass tubes.

Salt water is introduced to the cycle into the shell of the first effect evaporator with a concentration of 36,000 ppm. The resulting brine is transmitted to the shells of each succeeding effect and finally wasted to the main condenser water discharge tunnel with a concentration of 54,000 ppm.

Steam from the station evaporator is introduced into the tube bundle of first effect to heat the salt water and create steam to heat the second and third effects. Steam so produced is finally condensed at the evaporator condenser. Vapor created by pure condensate in the tube bundles of the second and third effects is also collected at the evaporator condenser. The result is about 50 gpm of fresh water with a purity of 50 ppm. A portion of this is drawn off and transmitted to the raw water storage tank for utility purposes and the remainder is piped to the station evaporator for redistilling. Pure condensate so created is drawn from the first effect tube bundle and transmitted to the main condenser at a purity of less than 1 ppm. At the main condenser it is introduced into the boiler feed water system.

Summary of the Fresh Water Supply

A comparison of the sources of water indicated that the Upper Salinas River Dam and the Sewage Treatment Plant should be eliminated because a firm supply could not be assured.

An economic comparison of Toro Creek Dam and the sea water evaporators indicated that the capital cost of the evaporators was about 1/3 of the dam but the annual cost was greater beyond an installation of 2 units; however, if the operation of the evaporators were limited to the production of boiler makeup water only and the utility water supplied by wells their installation would be economically favorable up to a six unit installation.

This latter scheme was finally adopted with a slight modification of the cycle as shown in figure 6. The pump bearing lubrication water requirements are to be fulfilled by using salt water. The additional capacity of the evaporators serve as a standby in the event of a failure in the wells or the bearing lubrication system.

Foundations

The Morro Bay Steam Plant is located immediately west of the town of Morro Bay on a low section of land which terminates in the promontory leading to the dacite sea-stack of Morro Rock. To the west, between the site and the Pacific Ocean are shifting sand dunes having an average height of more than 30 feet above the adjacent ground. Just to the south, and on the opposite side of Morro Rock from the ocean, is the navigation channel leading to Morro Bay, while to the east is a 60-foot high sandstone bluff consisting predominantly of poorly cemented reddish-brown fine to coarse sand. Originally, the low and flat area now occupied by the plant had an average elevation of 6 feet, U.S. Coast and Geodetic Survey datum. However, during 1942, when the site was developed as a U.S. Navy training base, the area was filled hydraulically with sand and gravel to an average elevation of 14 feet.

Subsurface exploration of the site was carried out during the early months of 1953 at which time 43 holes were drilled to depths varying between 60 to 101 feet below ground surface. The various soil strata were classified by field examination, supplemented by laboratory tests on representative undisturbed core samples.

In general, the top 54 to 82 feet of natural deposits underlying the hydraulic fill are of recent geological origin. The deposits appear to be lenticular and consist of alluvial soil types: fine sands, sandy and silty loams, clay loams, and clays, the latter being relatively compressible. These recent strata lie over geologically older deposits consisting of coarse sands and gravels which have high densities and strengths and are relatively incompressible.

Underlying this formation at varying depth is the sandstone and shale bedrock of the region. Groundwater level varied from approximately Elevation 11 feet in the north and east sections of the site to approximately Elevation 2 feet adjacent to the shoreline.

Because of the presence of compressible clay stratum at only moderate depth, it was necessary to provide a foundation for the power building that would keep to a minimum anticipated total subsidence and limit differential settlement over the base of the structure. To this end, studies were made of both piled and continuous mat foundations. As a result, a mat was adopted, as this was found to be not only more economical than a piled foundation but to offer the following design and construction advantages:

1) Provided a basement through which the intake pipelines could be run to the condensers and in which other equipment and services could be conveniently located thereby allowing more spacious arrangement on upper floors.
2) Cut down on construction time by eliminating period required for pile driving.
3) Further reduced construction time by permitting equipment, piping, and cable installation in the basement to proceed concurrently with erection of steelwork above.

The substructure as designed and built for the Morro Bay Plant is shown in corss-section in Figure 7. The 5-foot thick mat of reinforced concrete, founded at Elevation -2.0 feet, is 267 feet long by 186 feet - 6 inches wide. It was designed to act monolithically with the ground floor slab and beams at Elevation 15.0 feet, with the perimeter walls and piers transferring the shears. The rigidity of this 17-foot deep substructure will not only bridge localized areas of weakness in the supporting soils but will also assure minimum differential settlement due to either differences in soil pressures or variations in thickness of compressible layers. All superstructure steel columns are supported on the foundation piers at ground-floor level. The turbine-generator pedestals, because of magnitude of the machine loads and necessity of avoiding transfer of vibrations, are separated from the rest of the structure and are supported directly on the mat. Together, the dead and live loads of both superstructure and substructure produce soil pressures varying from 2500 to 2800 pounds per square foot. However, as a result of the removal of 16 feet of soil and the buoyant effect of the groundwater, the effective pressure on the subsoil is reduced by nearly 2000 pounds per square foot, varying from approximately 500 to 800 pounds per square foot.

After the magnitude of the foundation loads had been determined, studies were made to determine probable settlements. In order to arrive at these, it was necessary to consider the construction sequence. As the earth load is reduced during excavation for the substructure, the underlying clayey soils swell and, later, are recompressed by the addition of the building loads. Settlement of the structure is affected by the length of time the excavation remains open, the length of time that the dewatering system remains in operation, the rate at which loading is applied, and the relative rigidity of the mat. For the investigation of settlements, it was assumed that the excavation would remain open less than one month, that the dewatering period would be six months, and that the loads would be applied gradually at a uniform rate of increase over a period of 18 months.

As the structural behavior of the foundation would lie somewhere between that of a perfectly flexible without restraint insofar as differential

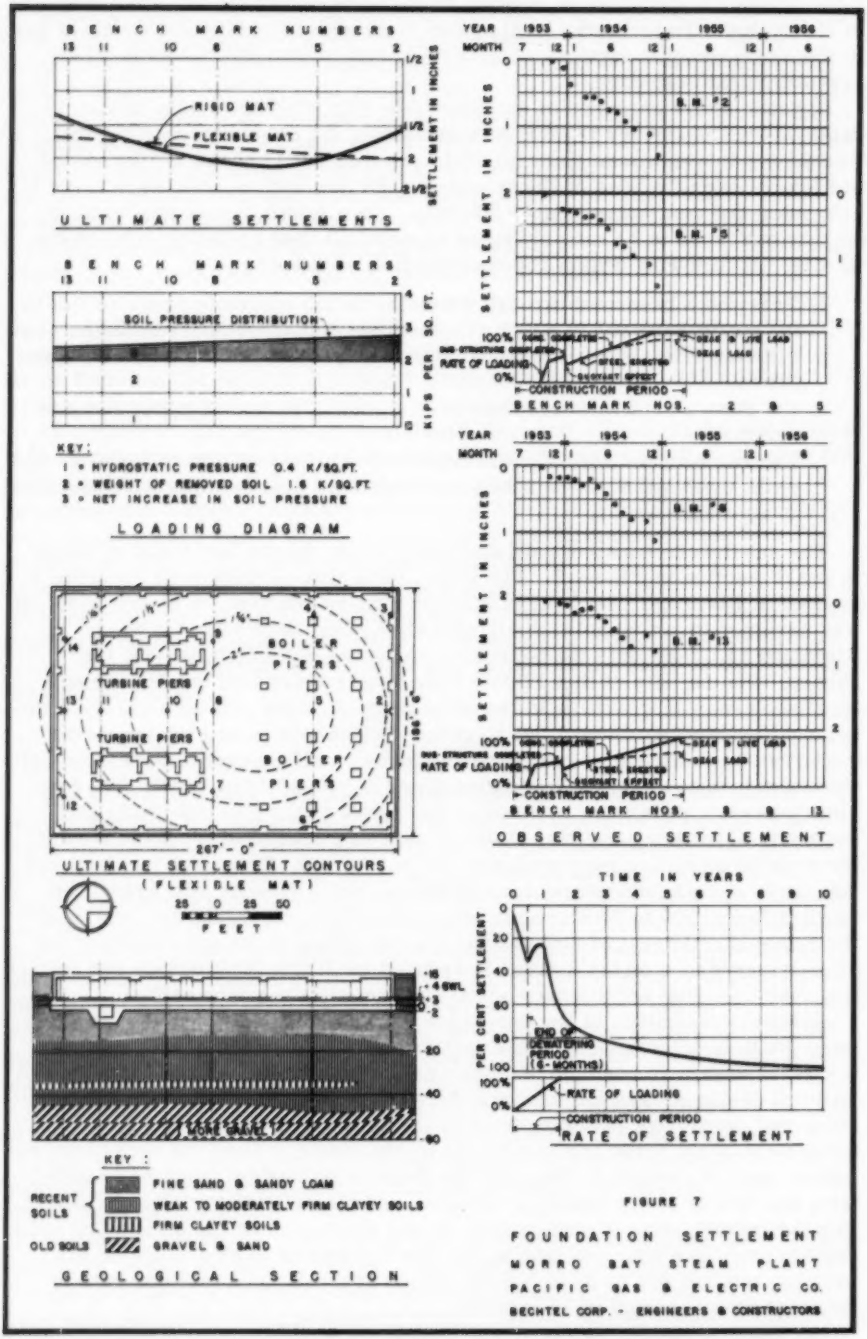

FIGURE 7

FOUNDATION SETTLEMENT

MORRO BAY STEAM PLANT

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737-19

are concerned and that of a perfectly rigid mat, allowing no warping of the mat surface, ultimate settlements were computed for these two limiting conditions. Results for the flexible mat are shown on Figure 7, Foundation Settlement, Morro Bay Steam Plant, over the titles of Ultimate Settlement Contours and Ultimate Settlements. In addition, the anticipated rate of vertical movement of any point on the mat as a function of time is given by the curve entitled Rate of Settlement. In this connection, it should be noted that almost 70 per cent of the total settlement is anticipated to take place during the construction period. It is expected that settlement at progressively decreasing rate will continue over a period of about ten years.

The accuracy of the computed settlements will depend on the validity of these assumptions. Actually, substructure excavation was started on July 3, 1953 and proceeded from the north end of the building under the turbine room to the south end under the boiler area. This was completed on August 28, 1953.

Construction of the mat also from north to south, began on August 17, 1953 and was completed on October 30, 1953. Both excavation and concreting proceeded in stages, rather than in a continuous operation.

The rate at which actual loading was applied was not uniform throughout the building. On completion of the concrete substructure, the steelwork was erected, again the turbine room first and the boiler support steel following later.

Erection of the first boiler located in the southeast quadrangle started in June 1954 and was completed in November 1954; work on the second boiler unit started immediately thereafter. The installation of the first turbine generator unit was scheduled to begin in January 1955.

Dewatering with well points was continuous for five months from July 27 to December 31, 1953, during which time pump discharge averaged 450 gpm. Construction of the cooling water intake lines and of the intake structure required dewatering to the south of the power building to continue about one year longer. The lowering of the water table there had probably an effect on the hydraulic gradient within the area of the powerhouse itself.

Settlement readings were taken regularly on a total of 14 permanent bench marks installed in the mat. Results for four of these points located on the N-S axis of the building are shown graphically on Figure 7 over the title of Observed Settlement. Initial and latest readings with their difference for all fourteen points are given below in Tables 1 and 2.

Mat Bench Mark Observations (in feet)

Bench Mark No.	1	2	3	4	5	6	7
Initial Reading September 1953	3.019	3.017	3.026	2.975	3.002	3.004	2.983
Latest Reading December 1954	2.915	2.895	2.883	lost	2.883	2.908	2.906
Difference	0.104	0.122	0.143	--	0.119	0.096	0.077

Table 1

Mat Bench Mark Observations (in feet)

Bench Mark No.	8	9	10	11	12	13	14
Initial Reading September 1953	2.966	3.013	2.998	2.954	2.970	2.999	2.999
Latest Reading December 1954	2.871	2.900	2.911	2.886	2.903	2.935	2.929
Difference	0.095	0.113	0.087	0.068	0.067	0.064	0.070

Table 2

From the records obtained during the first 15 months it appears that the distribution of settlements over the area of the structure is consistent with the pattern of applied loading. Maximum subsidence was measured in the southeast corner of the building where loading achieved 100 per cent of its ultimate value during the recorded period. The magnitude of settlements at the south end of the building may have been further affected by the dewatering operations in progress in the area south of the powerhouse as well as by the length of time the excavation remained open.

A study of relative settlements indicates that no noticeable warping of the surface occurred and that the assumption of a rigid mat would be correct.

Stacks

Although provisions have been made in the design of the two steam generators for future use of coal, normal fuel will be oil which will be burned at the rate of 250 barrels per hour per boiler at rated capability. Flue gases will be drawn from the two air preheaters at 330 deg. F and discharged through diametrically opposite openings into a single, 450 foot high stack, total volume of gases from both boilers at the same operating conditions being 1,100,000 cfm.

An inside diameter of 14 feet - 9 inches was selected for the stack top to give the gases an exit velocity of about 100 feet per second. The jet-like effect thus produced, combined with the temperature differential between the gases and the atmosphere, carries the plume an appreciable distance above the stack, adding to its effective height. In this connection, it should be noted that the dispersal and maximum ground concentration of gases varies inversely as the square of the effective stack height.

The reinforced concrete stack has an outside diameter at the base of 42 feet - 8 inches, and at the top of 17 feet - 6 inches, the shape tapering fairly uniformly. Shell thickness is dictated by structural requirements and varies from 24 inches at the base, to 27 inches at the flue openings, to 7 inches at the top. It is lined with acid-resisting firebrick and is insulated with 2-inch thick, glass fibre boards. The lower 105 feet of the brick lining is 9 inches thick and is designed to be self-supporting. Above this section, it is 4-1/2 inches thick and is supported at every 30 feet on concrete corbels formed monolithically with the shell.

The weight of the stack and lining and the overturning moments from both wind and seismic loadings are distributed to the foundation subsoils through a heavily reinforced concrete mat supported on 289 step-tapered concrete piles. This mat is octagonal in plan, having a dimension of 90 feet from face to face, and is 10-feet thick under the stack, the thickness decreasing to 6 feet - 6 inches at the mat periphery. The piles, carrying load by both end

bearing and friction, have an average length of 68 feet and are driven through the recent soils to a penetration of at least 3 feet into the dense, underlying sands and gravels of the older stratum. For dead load alone, maximum allowable load per pile was limited to 45 tons. For the condition of combined, vertical and seismic loadings, this was increased to 90 tons. It may be of interest to point out that the 2030 cubic yards of concrete required for construction of the pile cap was placed in a continuous operation lasting for 28 hours. Figure 8 shows the reinforcing steel for the foundation and dowels for the stack shaft.

Except for seismic loadings, structural design of the stack followed the recommendations of the Proposed Standard Specification for the Design and Construction of Reinforced Concrete Chimneys, dated 1953, prepared by Committee 505 of the American Concrete Institute.

Recent investigations have raised strong doubts regarding the validity of the heretofore widely used assumption that all parts of a structure are subjected to a uniform seismic acceleration during an earthquake. For this to be true, it is necessary that the structure be infinitely rigid. Because of the inherent flexibility of the average structure, designs based on the application of seismic forces which are the product of mass and constant acceleration will produce a structure that is weaker in its upper sections. This is especially true in the case of a relatively limber structure such as a tall stack, a fact which is borne out by a recent survey of damage to chimneys caused by the 1923 Kanto and the 1948 Fukui earthquakes in Japan[3]. Here it was found that out of 22 fractured stacks in only 5 cases was the point of fracture located below the third point of the height. It should be noted that since the 1923 eqrthquake, Japanese building codes have required that stacks be designed using a constant seismic coefficient of 0.15.

Here in America, two recently published codes for seismic design take cognizance of the fact that there is a functional relationship between height and the acceleration factor. One, prepared in 1951 by a Joint Committee composed of the San Francisco Section of the American Society of Civil Engineers and the Structural Engineers Association of Northern California, recommends that the total base shear should be distributed over the height of the structure in accordance with the following formula:

$$F_x = V \frac{W_x h_x}{\Sigma (Wh)} \quad \text{and} \quad V = C.W$$

where F_x is lateral force at the section under consideration

V is base shear

W_x is weight of the section under consideration

h_x is height of section above base of the stack

$\Sigma (Wh)$ is summation of the products of weight of each section and its height above the base.

The base shear is equal to the weight of the structure multiplied by a seismic coefficient.

3. The Fukui Earthquake—Vol. II prepared by the Office of the Engineer, GHQ, Far East Command—February, 1949.

The other, the Proposed Standard Specification for Design of Reinforced Concrete Chimneys, to which reference has already been made, recommends that a constant acceleration factor be used in the design of the lower one-fifth of the stack but that in the upper four-fifths, a magnification factor be applied to the seismic coefficient as determined by the empirical formula:

$$K_x = K_e \left(1 + \frac{h'}{100}\right)$$

where K_e is the seismic coefficient and h' is the distance from section under consideration to the section that is one-fifth of the total height of the chimney above the base.

In order to determine how these recommendations would affect the stack design, a trial section was selected, as shown in Figure 9 over the title Tentative Design, for which seismic shears and moments were calculated and analyzed. Included in this analysis were moments computed using a constant acceleration of 0.20.

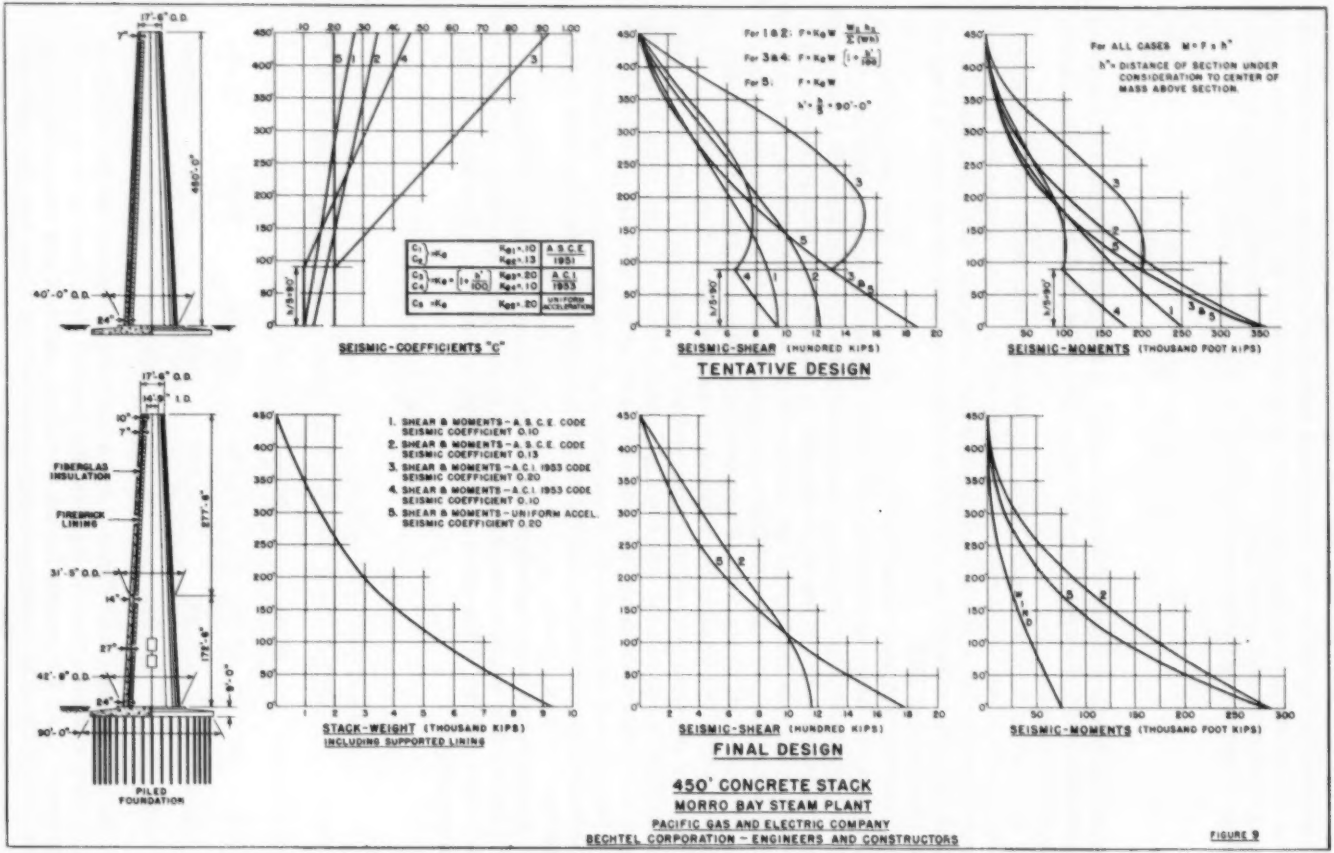
Two different base shear coefficients, 0.10 and 0.13, were used in the computations based on the A.S.C.E. Code. According to its provisions, for structures other than buildings, the Joint Committee recommends a maximum coefficient of 0.10. However, it was felt in this case that the moments, especially those near the base of the stack, should not be less than those produced by a constant acceleration of 0.20, the criterion established for design of all other structures at the site. From several trial calculations, it was determined that an increase in the value of the base shear coefficient to 0.13 would result in moments consistently greater than the required minimum. Moments calculated in accordance with the A.C.I. magnification factor and a 0.10 seismic coefficient were less than those based on 20 per cent acceleration in the bottom 175 feet of the stack. Consequently, it was necessary to use a coefficient of 0.20 in order to meet the moment requirement near the base of the stack.

Shear and moment curves prepared from these studies are shown on Figure 9. For comparison, the moments at the base, one-third, one-half, and two-third heights of the stack are tabulated below for the five design cases investigated:

Bending Moments (foot kips)

Case	1 ASCE, C = 0.10	2 ASCE, C = 0.13	3 ACI, K = 0.20	4 ACI, K = 0.10	5 K = 0.20
2/3h = 300'	28,000	36,000	86,000	43,000	26,000
1/2h = 225'	67,000	83,000	153,000	74,000	61,000
1/3h = 150'	120,000	152,000	196,000	98,000	128,000
Base	250,000	354,000	350,000	175,000	350,000

As will be seen, both Cases 2 and 3 meet the criterion that the resulting moments at all sections be consistently greater than those given by Case 5. However, Case 3 moments for the two-third, one-half and one-third heights are 230, 151 and 53 per cent greater than those for Case 5 at the same sections whereas those for Case 2 are only 38, 36, and 19 per cent greater than



450' CONCRETE STACK

MORRO BAY STEAM PLANT

PACIFIC GAS AND ELECTRIC COMPANY

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FIGURE 9

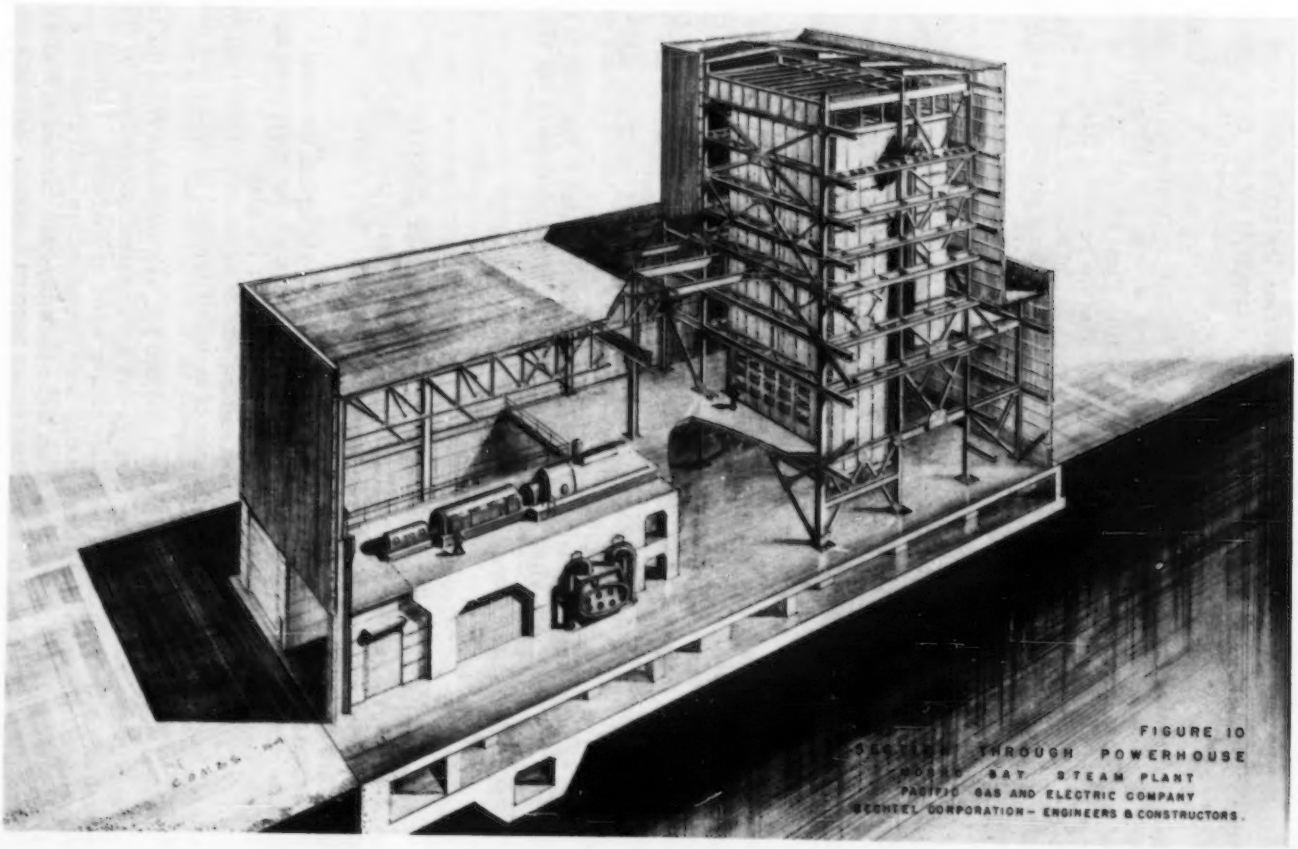

FIGURE 10
SECTION THROUGH POWERHOUSE
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the corresponding moments for Case 5. To accommodate these high moments for Case 3, a second trial design was prepared, the resulting gravity load further increasing final moments. For this condition, the stack shaft alone would require 2945 cubic yards of concrete, an increase of 30 per cent over the 2260 cubic yards required for one designed in accordance with Case 2.

Consistent with what was felt to be a sound engineering approach, the final stack design was prepared following the ASCE seismic recommendations using a base shear coefficient of 0.13. Stack outline, weight, and seismic shears and moments are shown on the figure over the title Final Design. It will be seen from the curves and from the table below that the resulting final moments, Case 2, are greater over the entire stack height and that they are considerably larger for the upper, critical sections than those produced by 20 per cent constant acceleration, Case 5.

Final Design - Moments (thousand foot-kips)

Height	0	50	100	150	200	250	300	350	400	450
Case 2	287	229	176	130	89	57	32	14	4	0
Case 5	283	205	144	97	62	37	20	8	1	0
Difference %	1.4	11.7	22.2	34.0	43.6	54.1	60.0	75.0	300.0	0

The fact that the stack weight and stress functions are smaller than those of the tentative design is attributable to certain changes introduced in the final design. These consisted of increasing the thickness of the bottom 105 feet of the lining to 9 inches and making it structurally self-supporting and adjusting the thickness of the concrete shell to actual stress requirements.

The construction of the stack shaft was started on July 12, 1954 and was completed on December 1, 1954.

It is believed that this stack of the Morro Bay Steam Plant is the first to be designed for seismic loading in accordance with recommendations of the 1951 Joint Committee.

Structural Features

Functionally the plant consists of three clearly defined elements: the boilerhouse, the turbine room and a firing aisle in between them. Structurally the three elements are designed to act as a single unit. See Figure 10, "Section Through Powerhouse."

Supported from a common reinforced concrete substructure, the steel superstructure was arranged to provide adequate support and convenient access to equipment as well as a weather enclosure for the plant.

Apart from vertical loading the structure was designed to withstand safely lateral forces caused by an earthquake of an intensity equal to twenty per cent of gravity.

The boilers, each weighing about 7,000,000 lbs. are suspended from the top of the boilerhouse steelwork by hangers—an arrangement necessitated by the 6-inch thermal expansion of the boiler casing while in operation. Lateral movements during an earthquake of this huge pendulum are restrained by a system of buckstays. Reactions from these buckstays are carried at 6 elevations by horizontal trusses to adjacent boiler support columns. Although oil is the only fuel at present available, provisions were made for future coal firing. In line with this, the end row of full height columns as well as the stub

columns were designed to support coal bunkers of 1200 tons capacity per unit.

A Warren-type system of bracing in both vertical planes provides stability of the structure for lateral forces. The roof framework and the concrete floors provide bracing in the horizontal plane. Column shafts were embedded to a depth of 3 feet in the concrete piers of the substructure. Vertical loading is transferred through the piers directly to the mat. Horizontal shears at column bases are distributed by the ground floor slab among the piers and from there carried to the foundation.

The turbine generators, supported on isolated pedestals, are set on the same axis as the steam generators. This arrangement of boilers and turbines allows for short runs of primary and reheat piping and also results in a minimum length of bus ducts between generator terminals and the station and main transformers. Such a plant layout, which afforded considerable overall economies, required an almost 120 feet wide turbine room unobstructed by intermediate columns. To bridge this span 12'- 6'' deep roof trusses were adopted. Although no portal frame action was considered when sizing the columns, a certain rigidity of connection between column and truss was assumed when designing the trusses.

The stepped columns of the turbine room carry, apart from the roof framework, crane girders for a 60-ton capacity bridge crane.

Bracing which is provided between the columns as well as between the trusses enables the turbine room structure to withstand seismic forces acting in a longitudinal direction. Transversely to the building, lateral loading is transferred to the boilerhouse structure by the combined action of the roof framework and concrete floors.

The operating controls of the plant are concentrated in the firing aisle. There, on the operating floor, is located the nerve center of the entire plant: the control room. Through its glassed walls, the operator must have an unobstructed view of the turbine ends, the boiler feed pumps and the various control panels. To achieve this, a 40 foot wide bay was required. (See Figures 11 and 12.) This was obtained by moving the turbine room and boilerhouse structures apart and providing horizontal framing which is supported by adjacent columns.

The 2-story office building, the machine shop and warehouse, although adjoining the power house, are structurally independent of it.

A total of 3950 tons of riveted structural steelwork was used in the construction of the plant.

Concrete

The problem of chemically reactive aggregates is common throughout Central California,[4] where sedimentary rocks of the opaline shale type form one of the sources of aggregate.

The outward manifestation of the presence of reactive aggregate is wide spread pattern cracking of the concrete. The mechanism of this alkali-aggregate reaction is a chemical process in which soluble alkalies released by the hydration of cement react with siliceous constituents of the aggregates to produce an alkali silica gel. Expansion of this gel which accompanies the process causes cracking and deterioration of the concrete.

4. "Alkali-Aggregate Reaction in California Concrete Aggregates," Report 27, Div. of Mines—State of California.

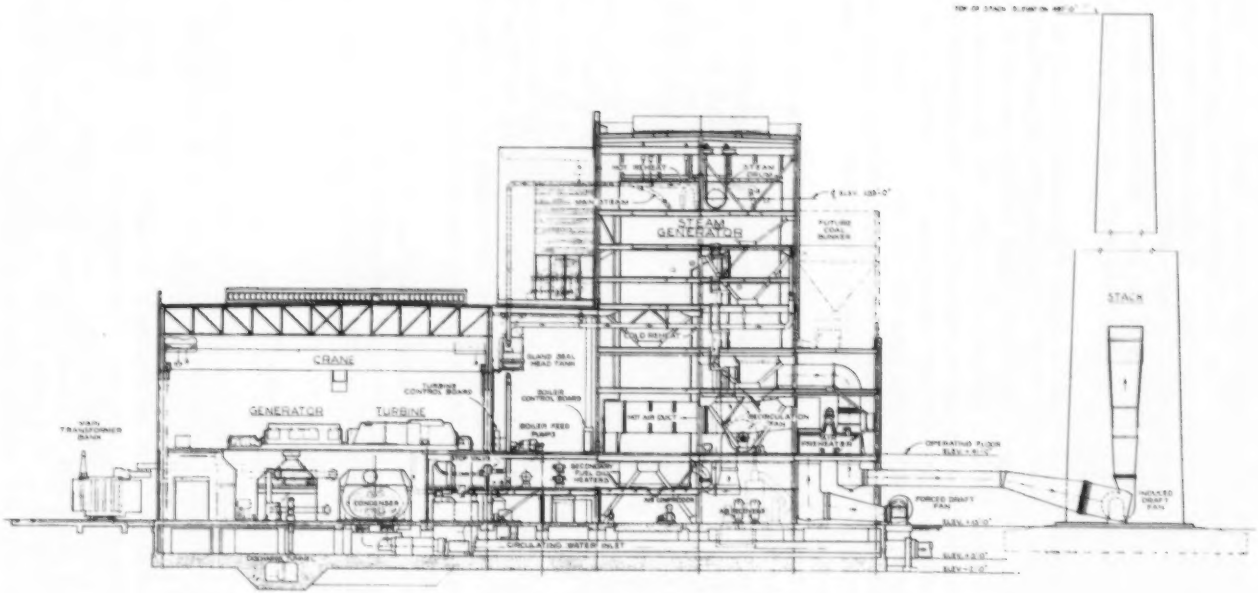

FIGURE 11

ELEVATION

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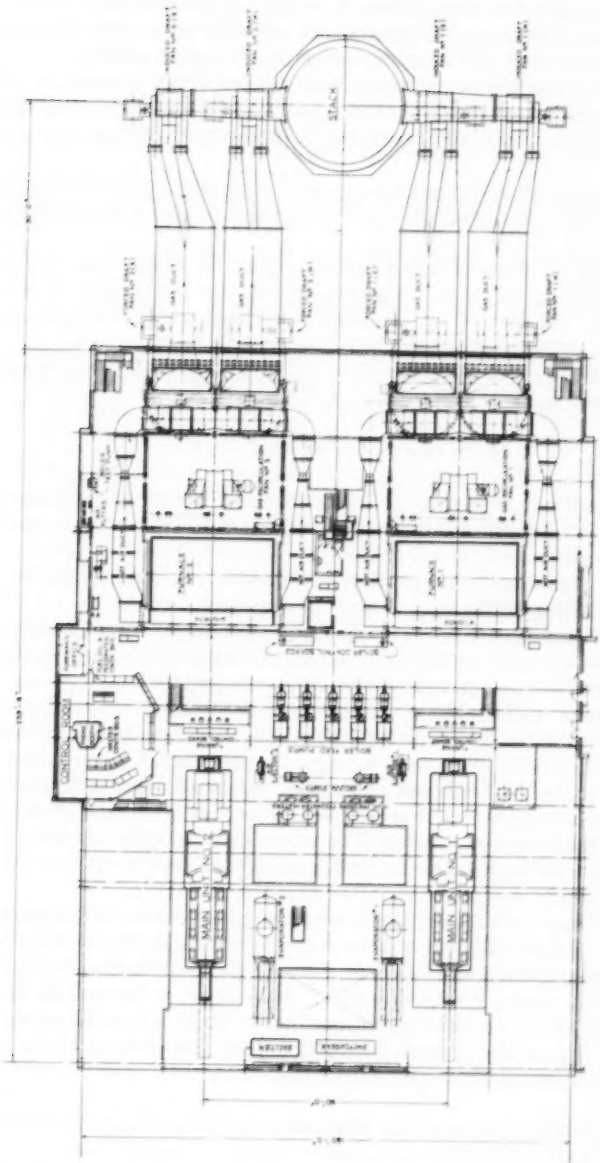

FIGURE 13
PLAN AT OPERATING FLOOR
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Extensive tests were carried out on aggregates from quarries in the vicinity of the plant. Coarse aggregate was required to show less than 12 per cent loss of weight after five cycles of sodium sulphate tests and the fine aggregates a loss of less than 8 per cent. As a further precautionary measure, 18 per cent of the cement was replaced by pozzolanic material,[5] a calcined diatomaceous earth, and low alkali Type II cement was used exclusively.

Although the design of concrete mixes was adjusted from time to time to meet particular needs, two of the mixes were used predominantly.

In heavy sections like the powerhouse mat and the stack foundation, where high strength was of secondary importance, a crushing strength at 28 days of 2500 psi was aimed at. This mix contained 4.75 sacks of cement per cu. yd. of concrete.

In all other structures a mix with a compressive strength of 3500 psi at 28 days was specified. The average cement content of this mix was 6 sacks per cu. yd.

All concrete was furnished from a batching plant of 75 cu. yds. per hour capacity installed at the jobsite. A careful scrutiny of the quality of concrete and aggregates was maintained by qualified personnel who had a fully equipped testing laboratory at their use.

A total of 41,500 cu. yds. of concrete was placed and 3,650 tons of reinforcing steels were used in the construction of the plant.

Turbine Generator Foundations[6]

The two turbine generator units of the Morro Bay Steam Plant have a rated capability of 156,250 kw which places them among the largest machines installed and in operation today.

These turbines are 3600 RPM, tandem-compound triple flow, and the generators are 3600 RPM, 18000 Volt, 3-phase, hydrogen-cooled.

The condensers are of the horizontal single pass type. Part of the weight of the condenser is carried on the turbine exhaust and the balance directly by the foundation through an arrangement of spring supports.

In view of the importance and the value of the equipment they carry, great care was exercised in the design and physical arrangement of the turbine generator foundation. This required a cooperative effort on the part of the mechanical and structural engineer. Based on information obtained from the turbine and condenser manufacturers, the former prepared an outline of the foundation and established its leading dimensions. It was the function of the structural engineer to analyze the behavior of the foundation under static and dynamic loading and to prepare a detailed structural design.

Static loading, impact factors, and maximum permissible deflections of members directly supporting the machine are specified by the turbine manufacturer. His recommendations include also allowable stresses to be used in the design and, as a guide, give the desired ratio between the weight of the machine and that of the concrete foundation. With these criteria established, the structural design of the turbine pedestal follows the pattern of analysis set up for rigid reinforced concrete frames.[7]

5. "Symposium on Use of Pozzolonic Materials in Concretes," Tech. Public. No. 99, ASTM
6. "Turbine Generator Foundations," GET-1749, General Electric Company
7. "Foundation for a Large Turbogenerator," Paul Rogers, ACI Journal, November, 1951.

The capability of turbine generators has increased sharply in recent years bringing in its wake a corresponding increase in weight and speed of machines. The need for a more rational approach to the problem of dynmaic design of turbine foundation has therefore become apparent.

Work of such investigators as Ehlers,[8] Kayser,[9] and Rausch,[10,11] have contributed much to the development of suitable methods of analysis.

The German Bureau of Standards recognizing this fact has included in a recently published specification[12] for the design and construction of turbine foundations a requirement for their dynamic investigations.

Provisions of this Specification cover the following main points:

1) Natural frequencies of the foundation must be calculated for both horizontal and vertical directions.
2) In order to avoid resonance natural frequencies of the structure should be either lower or higher than the speed of the unit. In either case the difference between the two frequencies should not be less than 20 per cent.
3) Natural frequencies of individual bents should be of the same order of magnitude.
4) The dynamic stability of the foundation can be assured by designing the structure for an equivalent static force. The magnitude of this static force depends on the weight of rotating parts of the machine and the ratio of natural to forced frequencies of the foundation.

In the design of turbine generator foundation of the Morro Bay Steam Plant the above criteria were incorporated.

Figure 13 shows the general arrangement of the foundation, the dynamic investigation of the foundation is given in Figure 14.

The turbine generators of the Morro Bay Steam Plant have not yet been set in service and consequently performance data of the pedestals are not available. However, the Pittsburg Steam Plant of the Pacific Gas and Electric Company has been in operation for quite some time. The turbine generators of both plants are of the same rated capacity, and the design of the foundation followed similar lines, with only adjustments to allow for condensers furnished by different manufacturers.

The performance record of the pedestals on the Pittsburg Plant is very satisfactory. The measured amplitudes of induced vibration of the foundation at deck level do not exceed 0.2 mils with a maximum amplitude recorded at the turbine bearing No. 3 inside of the exhaust hood, of 0.53 mils. The output of the machine when these readings were taken was 160,000.

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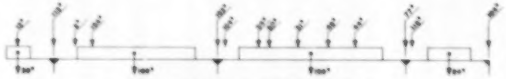
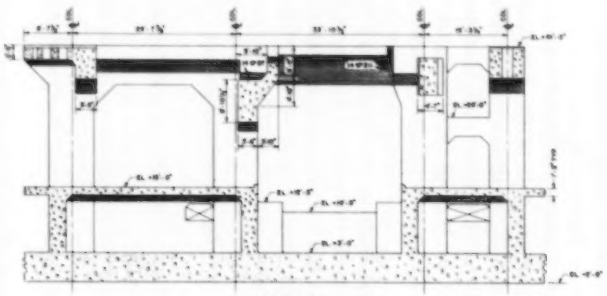
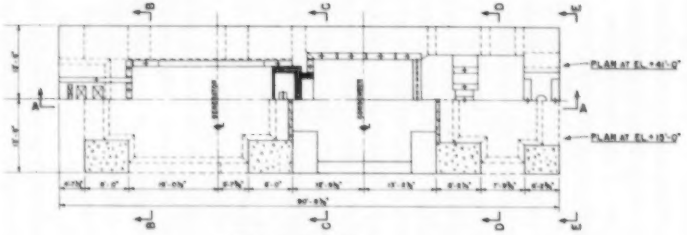
8. "Berechnungen der Schwingungen von Turbinenfundamente," G. Ehlers, Beton v Eisen, 1929.
9. "Theoretische Betrachtungen und Versuche über Fundamentschwingungen," Prof. H. Kayser, Beton v Eisen, 1930.
10. "Zur Berechnung von Dampfturbinenfundamenten"—Dr. E. Rausch—Beton v Eisen, 1931 and 1934.
11. "Maschinenfundamente und andere dynamischen Bauaufgaben"—Dr. E. Rausch, VDI Verlag—1940.
12. "Stuetzkonstruktionen fur Rotierende Maschinen, (Tischfundamente fur Dampfturbiene) Deutsche Industrie Normen—DIN 4024, December, 1954.



LOADING DIAGRAM FOR ONE LONGITUDINAL FRAME

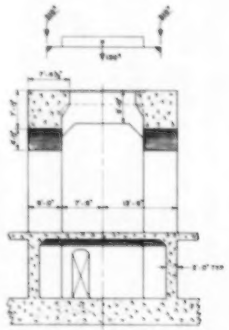


SECTION A-A

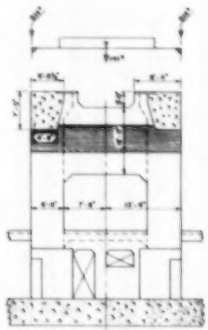


EQUIPMENT LOADS

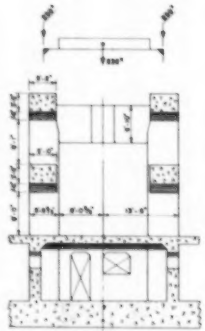
ALL EQUIPMENT LOADS ARE NET.
NO IMPACT FACTOR ALLOWED IN VIBRATION CALCULATIONS.
COLUMN LOADS INCLUDE HALF OF WEIGHT OF COLUMN.



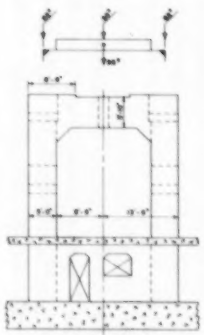
SECTION B-B



SECTION C-C



SECTION D-D



SECTION E-E

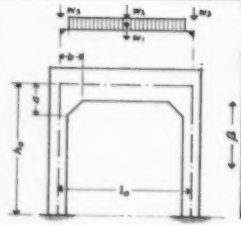
FIGURE 13

TURBINE AND GENERATOR FOUNDATION

MORRO BAY STEAM PLANT

PACIFIC GAS AND ELECTRIC COMPANY

BECHTEL CORPORATION - ENGINEERS AND CONSTRUCTORS

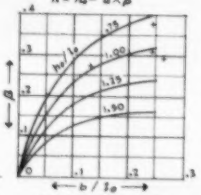


TYPICAL BENT

REDUCTION FORMULAS.

$l = l_o - 2b \times \beta$

$h = h_o - a \times \beta$



VALUES OF COEFFICIENT β

D E F I N I T I O N S .

H_o = horizontal force for unit deflection

M.C. = mass center (C.G. of W forces)

E.C. = elastic center (C.G. of H forces)

R_1 & R_2 = centers of rotation

F_y = deflection of E.C. for $H_o = 1$

f_z = rotation of z-axis for $M_o = 1$

h_n = distance y of BENT 'n' from E.C.

W, W_1 W_2 = dead load + equipment loads (mass loads) (W_2 includes 1/2 of column dead loads)

A = area of section

I = moment of inertia

$K = \frac{I_1}{I_2} \times \frac{h}{l}$

E_c = modulus of elasticity in compression = 620×10^3 K/ft^2 ($n = 7$)

E_s = modulus of elasticity in shear = $1/2 \times E_c$

$W_n = W_{n1} + W_{n2} + 2W_{n3}$

DEFLECTION FORMULAS.

$\Delta_1 = \frac{W_1 \times l^2}{384 \times E_c \times I_1} \times \frac{5K+2}{K+2}$ deflection due to moments

$\Delta_2 = \frac{W_2 \times l^2}{384 \times E_c \times I_2} \times \frac{8K+4}{K+2}$ deflection due to moments

$\Delta_3 = \frac{3}{5} \times \frac{(1/2 W_1 + W_2)}{E_c \times A_1} \times l$ deflection due to shear

$\Delta_4 = \frac{1/2 \times (W_1 + W_2) + W_3}{E_c \times A_4} \times h$ deflection due to column compression

VERTICAL VIBRATION.

$n_e = \frac{187.7}{\sqrt{\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4}}$ cycles / min. (deflection in inches)

BENT	h_n / l_o	a / b	h_n / l_o	β	h / l		A	I	$E_c \times A \times 10^{-3}$	$E_c \times I \times 10^{-3}$	K	W_1 / W_2	W_3	$W_1 \times l^2 / 384$	$5K+2 / K+2$	$\Delta_1 \times 10^4$	$W_2 \times l^2 / 384$	$8K+4 / K+2$	$\Delta_2 \times 10^4$	$\frac{1/2 W_1 + W_2}{2} \times l$	$\Delta_3 \times 10^4$	$\frac{W_1 + W_2}{E_c \times A_4} \times h$	$\Delta_4 \times 10^4$	$\sum \Delta \times 10^4$	n_e
	ft	ft			ft		ft²	ft⁴	K	K ft²		K	K	K ft²		in	K ft²		in	K ft	in	K	in	in	cycles/min
A	23.0	5.5	1.10	0.29	21.4	columns	33.3	113	231																
	21.0	5.5	0.38		17.0	beam	38.2	98	218	545	1.05	130 / 0	310	1909	2.36	9									
B	17.0	9.0	0.81	0.38	13.6	columns	48.0	148	248															8035	42
	21.0	9.0	0.14		17.2	beam	37.8	648	214	3896	5.49	141 / 0	516	1868	3.53	2								7939	32
C	20.4	3.4	0.95	0.27	16.5	columns	41.2	107	255															7128	4
	21.5	3.7	0.13		20.0	beam	31.9	118	195	744	1.17	232 / 0	290	4833	2.65	18								7949	32
D	23.0	9.7	1.09	0.30	21.3	columns	28.0	56	161															1592	9
	22.0	9.2	0.14		18.9	beam	37.2	117	231	646	2.34	46 / 38	92	1988	3.16	9	668	5.23	8	479	5	2748	20	7137	34
																								1592	9
																								7137	34
																								1592	9
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																								1592	9
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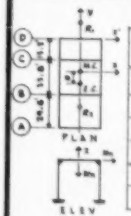
V E R T I C A L D E F L E C T I O N S & V I B R A T I O N S



PLAN

ELEV

BENT	W	y'	$W y' / 10^3$	v	$W y^2 / 10^3$	$GE_c I_o / h^3 \times 10^3$	$\frac{1+6.0K}{1+1.5K}$	H_o 10^3	$H_o y' / 10^3$	b_n	$H_o b_n^2 / 10^3$
	K	Ft.	Ft K	Ft.	Ft² K			Ft⁻¹ K		Ft	Ft K
A	750	78.7	590	-34.9	914	425	2.82	120	944	-34.3	1412
B	1169	49.1	574	-5.3	33	2129	3.92	749	3678	- 4.7	169
C	732	15.3	112	28.5	595	537	2.92	197	240	29.1	1329
D	262	0	0	43.8	503	208	3.33	69	0	44.4	1360
Σ	2913		1276		2045			1035	4862		4266
	Mass center	$y_m = 1276/29.13 = 43.8'$				Elastic center		$y_e = 4862/109.5 = 44.4'$			
		$y = 43.8 - y'$						$b = 44.4 - y'$			

HORIZONTAL DEFLECTIONS & VIBRATIONS

$H_n = \frac{GE_c}{h^3} \times I_{bn} \frac{1+6.0K}{1+1.5K}$ Ft⁻¹ K

$f_y = \frac{1}{\Sigma H_o} = \frac{10^3}{1095}$ ft K⁻¹

$f_z = \frac{1}{\Sigma H_o b n^2} = \frac{10^3}{4266}$ ft⁻¹K⁻¹

$f_y / f_z = 309.6$ ft²

$\frac{\Sigma W y^2}{\Sigma W} = 702.0$ ft²

$e = 0.6$ ft

$e^2 = 0.4$ ft²

$\phi = \left(a^2 + \frac{\Sigma W y^2}{\Sigma W}\right) \div \frac{f_y}{f_z} \times \frac{1}{\Sigma W y^2} = \frac{267}{10^3}$ K⁻¹

$\beta = \phi^2 - \frac{1}{\Sigma W \times \Sigma W y^2} \times \frac{f_y}{f_z} = \frac{9900}{10^{12}}$ K⁻²

$n_\alpha = \frac{187.7}{\sqrt{f_y}}$ $\sqrt{\phi \pm \sqrt{\beta}}$ cycles/min. (linear dimension in inches)

$n_\alpha = 187.7 \sqrt{1095 \times 10^3 \left(\frac{267}{10^3} \pm \frac{27}{10^3}\right)} \times \frac{1}{\sqrt{12}}$

Maximum vibration $n_{\alpha a} = 1091$ cycles/min. applies to R_1 .

Minimum vibration $n_{\alpha b} = 782$ cycles/min. applies to R_2 .

Rotation centers R_1 & R_2 :

$\lambda = \left(a^2 + \frac{\Sigma W y^2}{\Sigma W} \frac{f_z}{f_y}\right) \times \frac{1}{\Sigma a} = 360.7$ ft

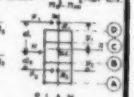
$y = \lambda - a \pm \sqrt{\lambda^2 + \frac{f_z}{f_y}}$ $\quad y_1 = 921.5$ ft (R_1) $\quad y_2 = -1.5$ ft (R_2)

SUMMARY: The system vibrates with frequencies $n_{\alpha a}$ & $n_{\alpha b}$ about centers R_1 & R_2 . Any periodic force H has parallel components F_1 & F_2 acting on R_1 & R_2 respectively.

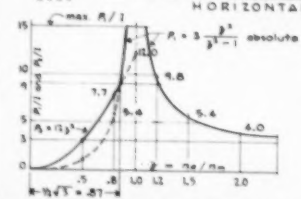
Equiv. static force $P = \frac{3 n_m^2}{n_\alpha^2 - n_m^2} \times \Sigma F$ cts.

$F_1 = H \times \frac{d_2}{d_1 + d_2}$

$F_2 = H \times \frac{d_1}{d_1 + d_2}$



PLAN



COEFFICIENT OF EQUIVALENT STATIC FORCES

E Q U I V A L E N T S T A T I C F O R C E S

n_e = natural frequency of foundation in cycles / m.n.

n_m = forced frequency in R. P. M. = 3600

L = weight of rotating parts

Tuning ratio $\beta = n_e / n_m = n_e / 3600$

Equivalent static coefficient ν

a) for $\beta < 1/2\sqrt{3}$ $\quad \nu_1 = 12 \beta^2$

b) for $\beta > 1/2\sqrt{3}$ $\quad \nu_2 = 3 \times \beta^2 / (\beta^2 - 1)$ with a max. of 15.0

Equivalent static force $P = \nu \times L$

vertically : max. $\beta = 3045 / 3600 = 0.85$ $\quad \nu_1 = 12 \times 0.85^2 = 8.7$

$P = 8.7 \times 170^K = 1479^K$

horizontally : max. $\beta = 1092 / 3600 = 0.29$ $\quad \nu_1 = 12 \times 0.29^2 = 1.00$

using minimum $\nu_1 = 5.0$

$P = 5.0 \times 170^K = 850^K$

WEIGHT OF MACHINE INCLUDING ROTATING PARTS

Turbine	621 Kips
Generator	617 "
Exciter	24 "
Total	1262 Kips

WEIGHT OF ROTATING PARTS

Turbine	74 Kips
Generator	96 "
Total	170 Kips

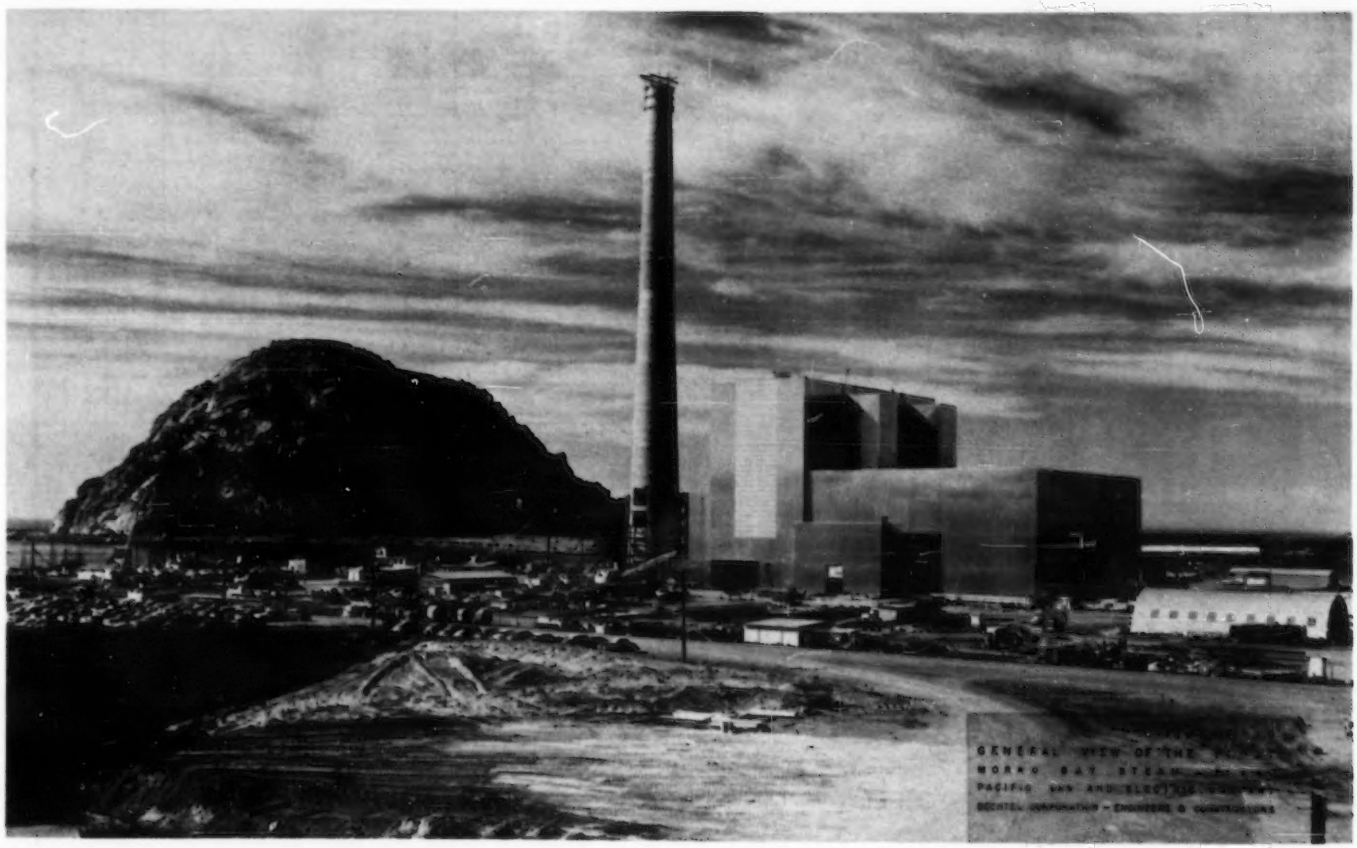
FIGURE 14

TURBINE & GENERATOR FOUNDATION DYNAMIC ANALYSIS

MORRO BAY STEAM PLANT

PACIFIC GAS & ELECTRIC CO.

BECHTEL CORP. — ENGINEERS & CONSTRUCTORS


GENERAL VIEW OF THE
MORRO BAY STEAM
PACIFIC GAS AND ELECTRIC
BECHTEL CORPORATION - ENGINEERS & CONSTRUCTORS

737-34

Architectural Features

The location of the plant near the City of Morro Bay and just off California's scenic Highway No. 1 prompted an architectural treatment, which while not obscuring the function of the structure, would at the same time be aesthetically pleasing. Although the climate is mild and suitable for outdoor operation the close proximity of the plant to the ocean combined with the humid atmosphere dictated the necessity for a totally enclosed plant.

A fluted aluminum siding of S-3 alloy was chosen for the walls of the power building. Such sheet siding provides flexibility in that it can be removed or re-used in the case of expansion, while providing a tight sheathing.

Service buildings, such as offices, machine shop, and warehouse are faced with porcelain-enamelled steel panelling, chosen for its resistance to atmospheric attack. The finish of this material allows a free choice of color, texture and size of panels. It provided contrast and distinguished the function of these smaller elements, avoiding the monotony that large areas of a single material would have otherwise produced

CONCLUSIONS

This paper has reviewed some of the problems encountered by the civil engineer as well as solutions adopted in the design of a large central steam electric station. It is hoped that the information contained therein will be of some benefit to the profession.

PROCEEDINGS PAPERS

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

VOLUME 80 (1954)

JULY: 457(AT), 458(AT), 459(AT)[c], 460(IR), 461(IR), 462(IR), 463(IR)[c], 464(PO), 465(PO)[c].

AUGUST: 466(HY), 467(HY), 468(ST), 469(ST), 470(ST), 471(SA), 472(SA), 473(SA), 474(SA), 475(SM), 476(SM), 477(SM), 478(SM)[c], 479(HY)[c], 480(ST)[c], 481(SA)[c], 482(HY), 483(HY).

SEPTEMBER: 484(ST), 485(ST), 486(ST), 487(CP)[c], 488(ST)[c], 489(HY), 490(HY), 491(HY)[c], 492(SA), 493(SA), 494(SA), 495(SA), 496(SA), 497(SA), 498(SA), 499(HW), 500(HW), 501(HW)[c], 502(WW), 503(WW), 504(WW)[c], 505(CO), 506(CO)[c], 507(CP), 508(CP), 509(CP), 510(CP), 511(CP).

OCTOBER: 512(SM), 513(SM), 514(SM), 515(SM), 516(SM), 517(PO), 518(SM)[c], 519(IR), 520(IR), 521(IR), 522(IR)[c], 523(AT)[c], 524(SU), 525(SU)[c], 526(EM), 527(EM), 528(EM), 529(EM), 530(EM)[c], 531(EM), 532(EM)[c], 533(PO).

NOVEMBER: 534(HY), 535(HY), 536(HY), 537(HY), 538(HY)[c], 539(ST), 540(ST), 541(ST), 542(ST), 543(ST), 544(ST), 545(SA), 546(SA), 547(SA), 548(SM), 549(SM), 550(SM), 551(SM), 552(SA), 553(SM)[c], 554(SA), 555(SA), 556(SA), 557(SA).

DECEMBER: 558(ST), 559(ST), 560(ST), 561(ST), 562(ST), 563(ST)[c], 564(HY), 565(HY), 566(HY), 567(HY), 568(HY)[c], 569(SM), 570(SM), 571(SM), 572(SM)[c], 573(SM)[c], 574(SU), 575(SU), 576(SU), 577(SU), 578(HY), 579(ST), 580(SU), 581(SU), 582(Index).

VOLUME 81 (1955)

JANUARY: 583(ST), 584(ST), 585(ST), 586(ST), 587(ST), 588(ST), 589(ST)[c], 590(SA), 591(SA), 592(SA), 593(SA), 594(SA), 595(SA)[c], 596(HW), 597(HW), 598(HW)[c], 599(CP), 600(CP), 601(CP), 602(CP), 603(CP), 604(EM), 605(EM), 606(EM)[c], 607(EM).

FEBRUARY: 608(WW), 609(WW), 610(WW), 611(WW), 612(WW), 613(WW), 614(WW), 615(WW), 616(WW), 617(IR), 618(IR), 619(IR), 620(IR), 621(IR)[c], 622(IR), 623(IR), 624(HY)[c], 625(HY), 626(HY), 627(HY), 628(HY), 629(HY), 630(HY), 631(HY), 632(CO), 633(CO).

MARCH: 634(PO), 635(PO), 636(PO), 637(PO), 638(PO), 639(PO), 640(PO), 641(PO)[c], 642(SA), 643(SA), 644(SA), 645(SA), 646(SA), 647(SA)[c], 648(ST), 649(ST), 650(ST), 651(ST), 652(ST), 653(ST), 654(ST)[c], 655(SA), 656(SM)[c], 657(SM)[c], 658(SM)[c].

APRIL: 659(ST), 660(ST), 661(ST)[c], 662(ST), 663(ST), 664(ST)[c], 665(HY)[c], 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

c. Discussion of several papers, grouped by Divisions.

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